DIVISION 7

SECTION 3107F - STRUCTURAL ANALYSIS AND DESIGN OF COMPONENTS

3107F.1 General

3107F.1.1 Purpose. This section establishes the minimum performance standards for structural components. Evaluation procedures for seismic performance, strength and deformation characteristics of concrete, steel and timber components are prescribed herein. Analytical procedures for structural systems are presented in Section 3104F.

3107F.1.2 Applicability. This section addresses MOTs constructed using the following structural components:

1. Reinforced concrete decks supported by batter and/or vertical concrete piles.
2. Reinforced concrete decks supported by batter and/or vertical steel piles, including pipe piles filled with concrete.
3. Reinforced concrete decks supported by batter and/or vertical timber piles.
4. Timber decks supported by batter or vertical timber, concrete, or steel pipe piles.

3107F.2 Concrete Deck with Concrete or Steel Piles

3107F.2.1 Component Strength. The following parameters shall be established in order to compute the component strength:

1. Specified concrete compressive strengths
2. Concrete and steel modulus of elasticity
3. Yield and tensile strength of mild reinforcing and prestressed steel and corresponding strains
4. Confinement steel strength and corresponding strains
5. Embedment length
6. Concrete cover
7. Yield and tensile strength of structural steel
8. Ductility

In addition, for "existing" components, the following conditions shall be considered:

9. Environmental effects, such as reinforcing steel corrosion, concrete spalling, cracking and chemical attack
10. Fire damage
11. Past and current loading effects, including overload, fatigue or fracture
12. Earthquake damage
13. Discontinuous components
14. Construction deficiencies

3107F.2.1.1 Material Properties. Material properties of existing components, not determined from testing procedures, and of new components, shall be established using the following methodology.

The strength of structural components shall be evaluated based on realistic upper bound estimates of material properties, except for non-ductile components, which shall be evaluated based on design material properties. The following values shall be substituted (Section 5.3 of [7.1] and p. 3-73 & 3-74 of [7.2]):

Non-ductile components (shear):

$$f'_c = 1.0\,f'_c \qquad (7\text{-}1a)$$
$$f_y = 1.0\,f_y \qquad (7\text{-}1b)$$
$$f_p = 1.0\,f_p \qquad (7\text{-}1c)$$

Other components (moment, axial):

$$f'_c = 1.3f'_c \qquad (7\text{-}2a)$$
$$f_y = 1.1f_y \qquad (7\text{-}2b)$$
$$f_p = 1.0\,f_p \qquad (7\text{-}2c)$$

Capacity protected members, such as pile caps and joints (maximum demand):

$$f'_c = 1.7f'_c \qquad (7\text{-}3a)$$
$$f_y = 1.3f_y \qquad (7\text{-}3b)$$
$$f_p = 1.1f_p \qquad (7\text{-}3c)$$

where:

f'_c = Compressive strength of concrete
f_y = Yield strength of steel
f_p = Yield strength of prestress strands

"Capacity Design" (Section 5.3 of [7.1]) ensures that the strength at protected locations are greater than the maximum feasible demand, based on realistic upper bound estimates of plastic hinge flexural strength. In addition, a series of pushover analyses using moment curvature characteristics of pile hinges may be required.

Alternatively, if a moment-curvature analysis is performed that takes into account the strain hardening of the steel, the demands used to evaluate the capacity protected components may be estimated by multiplying the moment-curvature values by 1.25.

Based on a historical review of the building materials used in the twentieth century, guidelines for tensile and yield properties of concrete reinforcing bars and the compressive strength of structural concrete have been established (see Tables 6-1 to 6-3 of FEMA 356 [7.3]. The values shown in these tables can be used as default properties, only if as-built information is not available and testing is not performed. The values in Tables 31F-7-1 and 31F-7-2, are adjusted according to equations (7-1) through (7-3).

3107F.2.1.2 Knowledge Factor (k). *Knowledge factor, k, shall be applied on a component basis.*

The following information is required, at a minimum, for a component strength assessment:

1. *Original construction records, including drawings and specifications.*
2. *A set of "as-built" drawings and/or sketches, documenting both gravity and lateral systems (subsection 3102F.1.5) and any post-construction modification data.*
3. *A visual condition survey, for structural components including identification of the size, location and connections of these components.*
4. *In the absence of material properties, values from limited in-situ testing or conservative estimates of material properties (Table 31F- 7-1 and 31F-7-2).*
5. *Assessment of component conditions, from an in-situ evaluation, including any observable deterioration.*
6. *Detailed geotechnical information, based on recent test data, including risk of liquefaction, lateral spreading and slope stability.*

The knowledge factor, k, is 1.0 when comprehensive knowledge as specified above is utilized. Otherwise, the knowledge factor shall be 0.75. Further guidance on the determination of the appropriate k value can be found in Table 2-1 of FEMA 356 [7.3].

3107F.2.2 Component Stiffness. *Stiffness that takes into account the stress and deformation levels experienced by the component shall be used. Nonlinear load-deformation relations shall be used to represent the component load-deformation response. However, in lieu of using nonlinear methods to establish the stiffness and moment curvature relation of structural components, the equations of Table 31F-7-3 may be used to approximate the effective elastic stiffness, EI_e, for lateral analyses (see subsection 3107F.5 for definition of symbols).*

3107F.2.3 Deformation Capacity of Flexural Members. *Stress-strain models for confined and unconfined concrete, mild and prestressed steel presented in subsection 3107F.2.4 shall be used to perform the moment-curvature analysis.*

The stress-strain characteristics of steel piles shall be based on the actual steel properties. If as-built information is not available, the stress-strain relationship may be calculated per subsection 3107F.2.4.2.

For concrete in-filled steel piles, the stress-strain model for confined concrete shall be in accordance with subsection 3107F.2.4.1.

Each structural component expected to undergo inelastic deformation shall be defined by its moment-curvature relation. The displacement demand and capacity shall be calculated per subsections 3104F.2 and 3104F.3, as appropriate.

The moment-rotation relationship for concrete components shall be derived from the moment-curvature analysis per subsection 3107F.2.5.4 and shall be used to determine lateral displacement limitations of the design. Connection details shall be examined per subsection 3107F.2.7.

3107F.2.4 Stress-Strain Models

3107F.2.4.1 Concrete. *The stress-strain model and terms for confined and unconfined concrete are shown in Figure 31F-7-1.*

3107F.2.4.2 Reinforcement Steel and Structural Steel. *The stress-strain model and terms for reinforcing and structural steel are shown in Figure 31F-7-2.*

TABLE 31F-7-1

COMPRESSIVE STRENGTH OF STRUCTURAL CONCRETE (PSI)[1]

Time Frame	*Piling*	*Beams*	*Slabs*
1900-1919	*2,500-3,000*	*2,000-3,000*	*1,500-3,000*
1920-1949	*3,000-4,000*	*2,000-3,000*	*2,000-3,000*
1950-1965	*4,000-5,000*	*3,000-4,000*	*3,000-4,000*
1966-present	*5,000-6,000*	*3,000-5,000*	*3,000-5,000*

1. Concrete strengths are likely to be highly variable for an older structure

TABLE 31F-7-2

TENSILE AND YIELD PROPERTIES OF REINFORCING BARS FOR VARIOUS ASTM SPECIFICATIONS AND PERIODS (after Table 6-2 of [7.3])

				Structural[1]	Intermediate[1]	Hard[1]			
			Grade	**33**	**40**	**50**	**60**	**70**	**75**
			Minimum Yield[2] (psi)	**33,000**	**40,000**	**50,000**	**60,000**	**70,000**	**75,000**
ASTM	**Steel Type**	**Year Range[3]**	**Minimum Tensile[2] (psi)**	**55,000**	**70,000**	**80,000**	**90,000**	**80,000**	**100,000**
A15	Billet	1911-1966		X	X	X			
A16	Rail[4]	1913-1966				X			
A61	Rail[4]	1963-1966					X		
A160	Axle	1936-1964		X	X	X			
A160	Axle	1965-1966		X	X	X	X		
A408	Billet	1957-1966		X	X	X			
A431	Billet	1959-1966							X
A432	Billet	1959-1966					X		
A615	Billet	1968-1972			X		X		X
A615	Billet	1974-1986			X		X		
A615	Billet	1987-1997			X		X		X
A616	Rail[4]	1968-1997				X	X		
A617	Axle	1968-1997			X		X		
A706	Low-Alloy[5]	1974-1997						X	
A955	Stainless	1996-1997			X		X		X

General Note: An entry "X" indicates that grade was available in those years.

Specific Notes:
1. The terms structural, intermediate, and hard became obsolete in 1968.
2. Actual yield and tensile strengths may exceed minimum values
3. Until about 1920, a variety of proprietary reinforcing steels were used. Yield strengths are likely to be in the range from 33,000 psi to 55,000 psi, but higher values are possible. Plain and twisted square bars were sometimes used between 1900 and 1949
4. Rail bars should be marked with the letter "R."
5. ASTM steel is marked with the letter "W"

3107F.2.4.3 Prestressed Steel. The stress-strain model of Blakeley and Park [7.4] may be used for prestressed steel. The model and terms are illustrated in Figure 31F-7-3.

3107F.2.4.4 Alternative Stress-Strain Models. Alternative stress-strain models are acceptable if adequately documented and supported by test results, subject to Division approval.

3107F.2.5 Concrete Piles

3107F.2.5.1 General. The capacity of concrete piles is based on permissible concrete and steel strains corresponding to the desired performance criteria.

Different values may apply for plastic hinges forming at in-ground and pile-top locations. These procedures are applicable to circular, octagonal, rectangular, and square pile cross sections.

TABLE 31F-7-3 ***EFFECTIVE ELASTIC STIFFNESS***	
Concrete Component	EI_e/EI_g
Reinforced Pile	$0.3 + N/(f'_c A_g)$
Pile/Deck Dowel Connection[1]	$0.3 + N/(f'_c A_g)$
Prestressed Pile[1]	$0.6 < EI_e/EI_g < 0.75$
Steel Pile	***1.0***
Concrete w/ Steel Casing	$(E_s I_s + 0.25 E_c I_c)/E_s I_s + E_c I_c$
Deck	***0.5***

1 The pile/deck connection and prestressed pile may also be approximated as one member with an average stiffness of 0.42 EI_e/EI_g (Ferritto et al, 1999 [7.2])
N = is the axial load level.
E_S = Young's modulus for steel
I_S = Moment of inertia for steel section
E_C = Young's modulus for concrete
I_C = Moment of inertia for uncracked concrete section

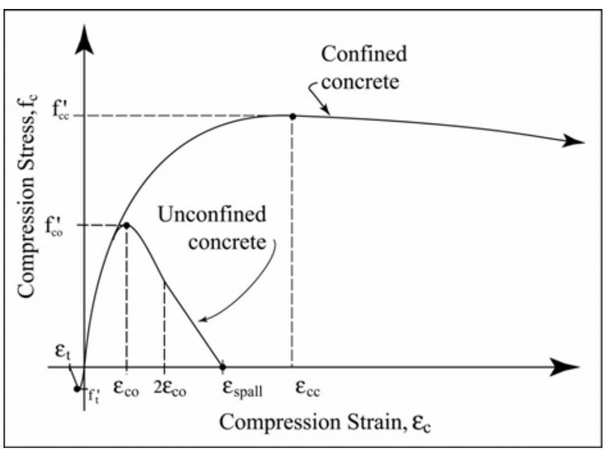


Figure 31F-7-1: Stress-Strain Curves for Confined and Unconfined Concrete [7.1]

3107F.2.5.2 Stability. *Stability considerations are important to pier-type structures. The moment-axial load interaction shall consider effects of high slenderness ratios (kl/r). An additional bending moment due to axial load eccentricity shall be incorporated unless:*

$$e/h \leq 0.10 \qquad (7\text{-}4)$$

where:

e = *eccentricity of axial load*

h = *width of pile in considered direction*

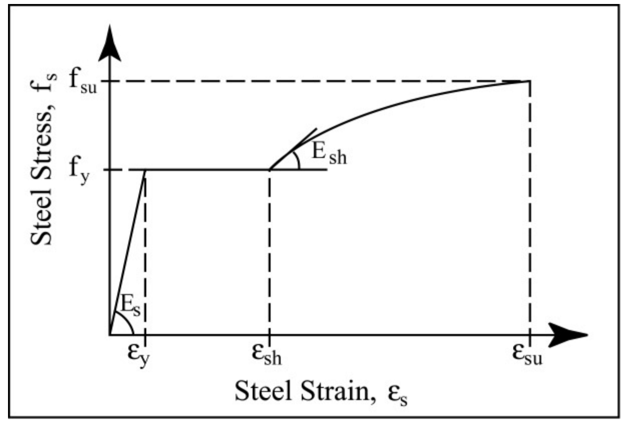


Figure 31F-7-2 Stress-Strain Curve for Mild Reinforcing Steel or Structural Steel [7.1]

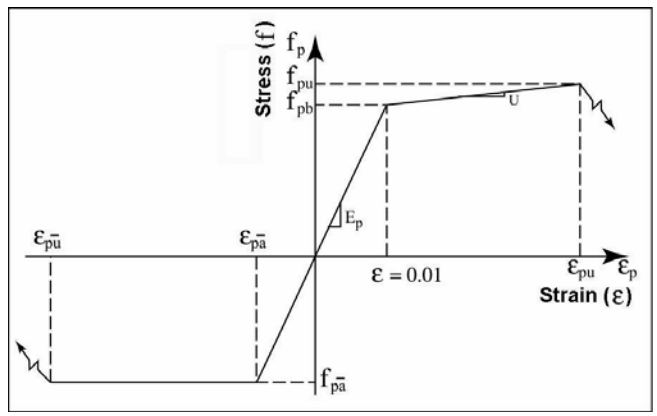


Figure 31F-7-3 Stress-Strain Curve for Prestressed Steel [7.4]

3107F.2.5.3 Plastic Hinge Length. *The plastic hinge length is required to convert the moment-curvature relationship into a moment-plastic rotation relationship for the nonlinear pushover analysis.*

The pile's plastic hinge length, L_p (above ground), when the plastic hinge forms against a supporting member is:

$$L_p = 0.08L + 0.15 f_{ye} d_{bl} \geq 0.3 f_{ye} d_{bl} \qquad (7\text{-}5)$$

where:

L = *the distance from the critical section of the plastic hinge to the point of contraflexure*

d_{bl} = *the diameter of the longitudinal reinforcement*

f_{ye} = *design yield strength of longitudinal reinforcement (ksi)*

If a large reduction in moment capacity occurs due to spalling, then the plastic hinge length shall be:

$$L_p = 0.3 f_{ye} d_{bl} \qquad (7\text{-}6)$$

When the plastic hinge forms in-ground, the plastic hinge length may be determined from Figure 31F-7-4 (see page 311 of [7.1]).

The stiffness parameter (x-axis) is:

$$\frac{KD^6}{[D^*]EI_e} \qquad (7\text{-}7)$$

where:

EI_e = *the effective stiffness*
K = *the subgrade modulus*
D = *pile diameter*
D^* = *reference diameter of 6 ft*

If site specific soil information is not available then the values for K in Table 31F-7-4 may be used.

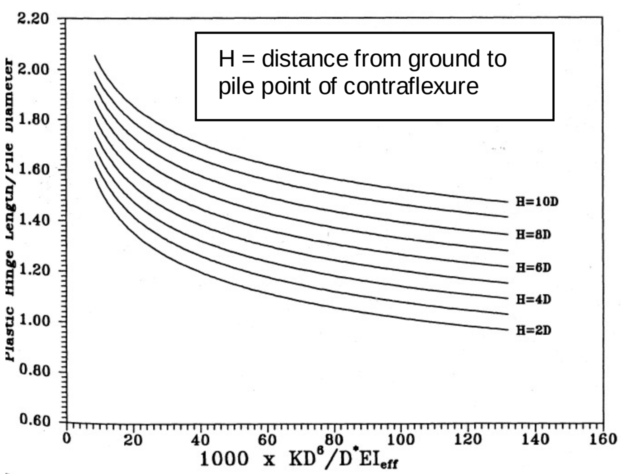


Figure 31F-7-4: Influence of Pile/Soil Stiffness Ratio on Plastic Hinge Length (after Fig 5.30 of [7.1])

3107F.2.5.4 Plastic Rotation. *The plastic rotation, θ_P, can be determined from Equation 31F-7-8, by using moment-curvature analysis and applicable strain limitations, as shown in Figure 31F-7-5.*

The plastic rotation is:

$$\theta_p = L_p \phi_p = L_p(\phi_m - \phi_y) \qquad (7\text{-}8)$$

where:

L_p = *plastic hinge length*
ϕ_p = *plastic curvature*
ϕ_m = *maximum curvature*
ϕ_y = *yield curvature*

TABLE 31F-7-4 *SUBGRADE MODULUS K*		
Soil Type	*Avg Undrained Shear Strength [psf]*	*Subgrade Modulus K [lb/in^3]*
Soft Clay	*250-500*	*30*
Medium Clay	*500-1000*	*100*
Stiff Clay	*1000-2000*	*500*
Very Stiff Clay	*2000-4000*	*1000*
Hard Clay	*4000-8000*	*2000*
Loose Sand (above WT/submerged)	-	*25/20*
Medium Sand (above WT/submerged)	-	*90/60*
Dense Sand (above WT/submerged)	-	*275/125*

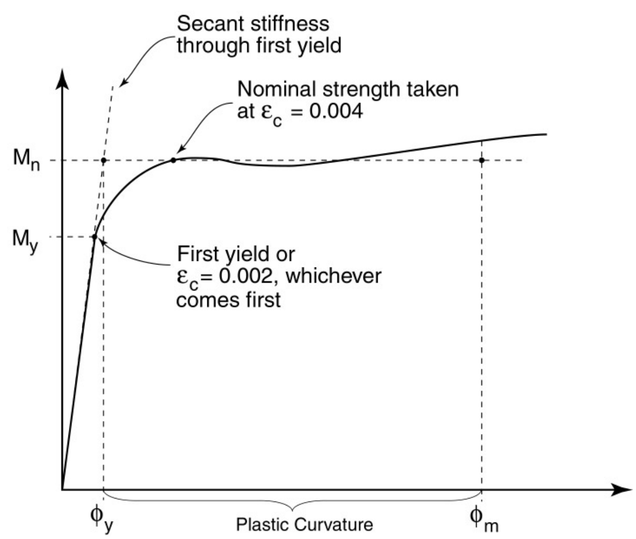


Figure 31F-7-5: Moment Curvature Analysis

The maximum curvature, ϕ_m, shall be determined by the concrete or steel strain limit state at the prescribed performance level, whichever comes first.

Alternatively, the maximum curvature, ϕ_m, may be calculated as:

$$\phi_m = \frac{\varepsilon_{cm}}{c_u} \qquad (7\text{-}9)$$

where:

ε_{cm} = *max limiting compression strain for the prescribed performance level (Table 31F-7-5)*

c_u = *neutral-axis depth, at ultimate strength of section*

The yield curvature, ϕ_y *is the curvature at the intersection of the secant stiffness,* EI_c*, through first yield and the nominal strength,* (ε_c = 0.004)

$$\phi_y = \frac{M_y}{EI_c} \qquad (7\text{-}10)$$

3107F.2.5.5 Ultimate Concrete and Steel Flexural Strains. *Strain values computed in the nonlinear pushover analysis shall be compared to the following limits for flexure:*

3107F2.5.5.1 Unconfined concrete piles*: An unconfined concrete pile is defined as a pile having no confinement steel or one in which the spacing of the confinement steel exceeds 12 inches.*

Ultimate concrete compressive strain:

$$\varepsilon_{cu} = 0.005 \qquad (7\text{-}11)$$

3107F.2.5.5.2 Confined concrete piles *[7.1]:*

Ultimate concrete compressive strain:

$$\varepsilon_{cu} = 0.004 + (1.4\rho_s f_{yh} \varepsilon_{sm})/f'_{cc} \geq 0.005 \qquad (7\text{-}12)$$

$$\varepsilon_{cu} \leq 0.035$$

where:

ρ_s = *effective volume ratio of confining steel*
f_{yh} = *yield stress of confining steel*
ε_{sm} = *strain at peak stress of confining reinforcement, 0.15 for grade 40, 0.12 for grade 60 and 0.10 for A82 grade 70 plain spiral*

f'_{cc} = *confined strength of concrete approximated by* $1.5 f'_c$

317F.2.5.6 Component Acceptance/Damage Criteria. *The maximum allowable concrete strains may not exceed the ultimate values defined in Section 3107F.2.5.5. The following limiting values (Table 31F-7-5) apply for each performance level for both existing and new structures. The "Level 1 or 2" refer to the seismic performance criteria (see subsection 3104F.2.1).*

For all non-seismic loading combinations, concrete components shall be designed in accordance with the ACI requirements [7.5].

Note that for existing facilities, the pile/deck hinge may be controlled by the capacity of dowel reinforcement in accordance with subsection 317F.2.7.

TABLE 31F-7-5

LIMITS OF STRAIN

Component Strain	*Level 1*	*Level 2*
MCCS Pile/deck hinge	$\varepsilon_c \leq 0.005$	$\varepsilon_c \leq 0.025$
MCCS In-ground hinge	$\varepsilon_c \leq 0.005$	$\varepsilon_c \leq 0.008$
MRSTS	$\varepsilon_s \leq 0.01$	$\varepsilon_s \leq 0.05$
MPSTS In-ground hinge	$\varepsilon_p \leq 0.005$ *(incremental)*	$\varepsilon_p \leq 0.04$ *(total strain)*

MCCS = Maximum Concrete Compression Strain, ε_c
MRSTS = Maximum Reinforcing Steel Tension Strain, ε_s
MPSTS = Maximum Prestressing Steel Tension Strain, ε_p

317F.2.5.7 Shear Capacity (Strength). *Shear strength shall be based on nominal material strengths, and reduction factors according to ACI-318 [7.5].*

To account for material strength uncertainties, maximum shear demand, $V_{max,push}$ *established from nonlinear pushover analyses shall be multiplied by 1.4 (Section 8.16.4.4.2 of ATC-32 [7.6]):*

$$V_{design} = 1.4V_{max,push} \qquad (7\text{-}13)$$

If moment curvature analysis that takes into account strain-hardening, an uncertainty factor of 1.25 may be used:

$$V_{design} = 1.25V_{max,push} \qquad (7\text{-}14)$$

If the factors defined in Section 31F-7.2.1.1 are used, the above uncertainty factors need not be applied.

As an alternative, the method of Kowalski and Priestley [7.7] may be used. This is based on a three-parameter model with separate contributions to shear strength from concrete (V_c*), transverse reinforcement (*V_s*), and axial load (*V_p*) to obtain nominal shear strength (*V_n*):*

$$V_n = V_c + V_s + V_p \qquad (7\text{-}15)$$

A shear strength reduction factor of 0.85 shall be applied to the nominal strength, V_n *, to determine the design shear strength. Therefore:*

$$V_{design} \leq 0.85\, V_n \qquad (7\text{-}16)$$

The equations to determine V_c, V_s and V_p are:

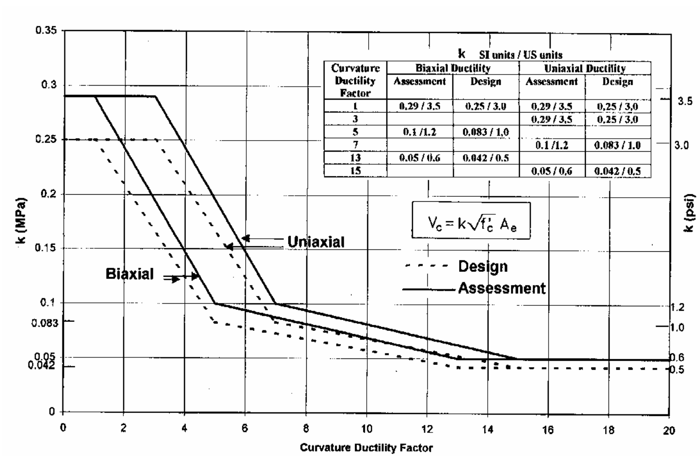
$$V_c = k\sqrt{f_c^{'}}A_e \tag{7-17}$$

where:

k = *factor dependent on the curvature ductility $\mu_\phi = \phi / \phi_y$, within the plastic hinge region, from Figure 31F-7-6. For regions greater than $2D_p$ (see eqn. 7-18) from the plastic hinge location, the strength can be based on $\mu_\phi = 1.0$ (see Ferritto et. al.[7.2]).*

$f_c^{'}$ = *concrete compressive strength*

A_e = *$0.8A_g$ is the effective shear area*



Curvature Ductility Factor	Biaxial Ductility Assessment	Biaxial Ductility Design	Uniaxial Ductility Assessment	Uniaxial Ductility Design
1	0,29 / 3,5	0.25 / 3.0	0,29 / 3.5	0,25 / 3,0
3			0,29 / 3,5	0,25 / 3.0
5	0.1 /1.2	0,083 / 1,0		
7			0.1 /1.2	0.083 / 1.0
13	0.05 / 0.6	0.042 / 0.5		
15			0,05 / 0,6	0.042 / 0,5

31F Figure 7-6: Concrete shear Mechanism (from Fig 3-30 of [7.1])

Circular spirals or hoops [7.2]:

$$V_s = \frac{\frac{\pi}{2} A_{sp} f_{yh} \left(D_p - c - c_o\right) cot(\theta)}{s} \tag{7-18}$$

where:

A_{sp} = spiral or hoop cross section area

f_{yh} = yield strength of transverse or hoop reinforcement

D_p = pile diameter or gross depth (in case of a rectangular pile with spiral confinement)

c = *depth from extreme compression fiber to neutral axis (N.A.) at flexural strength (see Fig. 31F-7-7)*

c_o = *concrete cover to center of hoop or spiral (see Fig. 31F-7-7)*

θ = *angle of critical crack to the pile axis (see Fig. 31F-7-7) taken as 30° for existing structures, and 35° for new design*

s = spacing of hoops or spiral along the pile axis

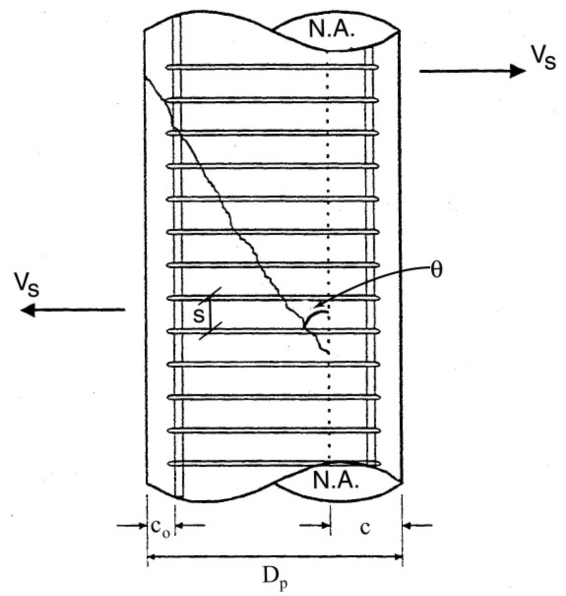


Figure 31F-7-7 Transverse Shear Mechanism

Rectangular hoops or spirals [7.2]*:*

$$V_s = \frac{A_h f_{yh} \left(D_p - c - c_o\right) cot(\theta)}{s} \tag{7-19}$$

where:

A_h = *total area of transverse reinforcement, parallel to direction of applied shear cut by an inclined shear crack*

Shear strength from axial mechanism, V_p (see Fig. 31F-7-8):

$$V_p = \Phi\left(N_u + F_p\right)\tan\alpha \tag{7-20}$$

where:

N_u = *external axial compression on pile including seismic load. Compression is taken as positive; tension as negative.*

F_p = *prestress compressive force in pile*

α = *angle between line joining centers of flexural compression in the deck/pile and in-ground* hinges, and the pile axis

Φ = 1.0 for existing structures, and 0.85 for new design

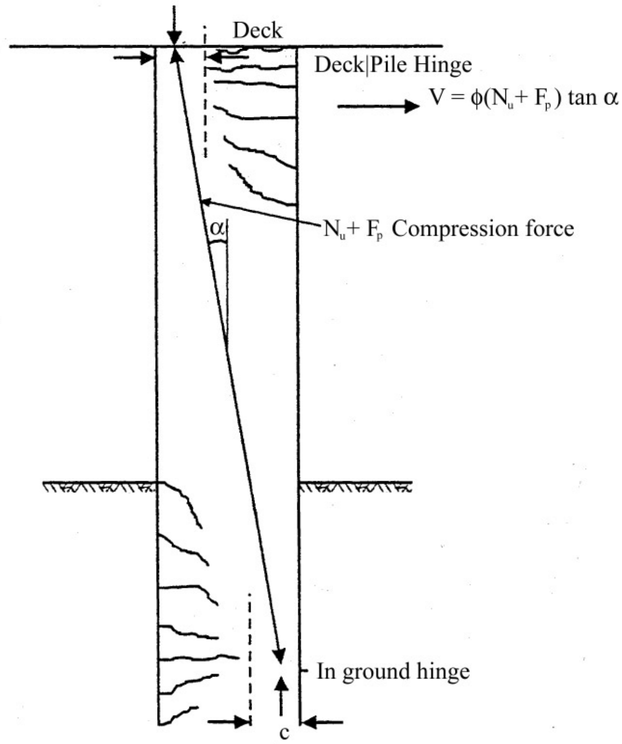


Figure 31F-7-8: Axial Force Shear Mechanism

3107F.2.6 Steel Piles

3107F.2.6.1 General. *The capacity of steel piles is based on allowable strains corresponding to the desired performance criteria and design earthquake.*

3107F.2.6.2 Stability. *Subsection 3102F.2.5.2 applies to steel piles.*

3107F.2.6.3 Plastic Hinge Length. *The plastic hinge length depends on the section shape and the slope of the moment diagram in the vicinity of the plastic hinge.*

For plastic hinges forming in steel piles at the deck/pile interface and where the hinge forms in the steel section rather than in a special connection detail (such as a reinforced concrete dowel connection), allowance should be made for strain penetration into the pile cap. This increase may be taken as $0.25D_p$, where D_p is the pile diameter or pile depth in the direction of the applied shear force.

3107F.2.6.4 Ultimate Flexural Strain Capacity. *The following limiting value applies:*

Strain at extreme-fiber, $\varepsilon_u \leq 0.035$

3107F.2.6.5 Component Acceptance/Damage Criteria. *The maximum allowable strain may not exceed the ultimate value defined in subsection 3107F.2.6.4. Table 31F-7-6 provides limiting strain values for each performance level, for both new and existing structures.*

TABLE 31F-7-6

STRUCTURAL STEEL STRAIN LIMITS, ε_u

Components	*Level 1*	*Level 2*
Concrete Filled Pipe	*0.008*	*0.030*
Hollow Pipe	*0.008*	*0.025*

Level 1 or 2 refer to the seismic performance criteria (subsection 3104F.2.1)

Steel components for all non-seismic loading combinations shall be designed in accordance with AISC-LRFD [7.8].

3107F.2.6.6 Shear Capacity (Strength). *The procedures of subsection 3107F.2.5.7 to establish V_{design} are applicable to steel piles (Equations 7-13 and 7-14). If the factors defined in subsection 3107F.2.1.1 are used, the uncertainty factors need not be applied.*

The shear capacity shall be established from the AISC-LRFD [7.8]. For concrete filled pipe, equation 7-15 may be used to determine shear capacity, however V_{shell} must be substituted for V_s; it thus becomes:

$$V_{shell} = (\pi/2)\, t\, f_{y,shell}\, (D_p - c - c_0) \cot\theta \qquad (7\text{-}21)$$

where:

t = shell thickness

$f_{y,shell}$ = yield strength of steel shell

c_o = outside of steel pipe to center of hoop or spiral

(All other terms are as listed for equation 7-18).

3107F.2.7 Pile/Deck Connection Strength

3107F.2.7.1 Joint Shear Capacity. *The joint shear capacity shall be computed in accordance with ACI 318 [7.5]. For existing MOTs, the method [7.1, 7.2] given below may be used:*

1. *Determine the nominal shear stress in the joint region corresponding to the pile plastic moment capacity.*

$$v_j = \frac{0.9 M_p}{\sqrt{2}\, l_{dv} D_p^2} \qquad (7\text{-}22)$$

where:

ν_j = *Nominal shear stress*

M_p = *Overstrength moment of the plastic hinge (the maximum possible moment in the pile) as determined from a pushover analysis at displacements corresponding to the damage control limit state (1.25 M_n when established from moment curvature and 1.3 and 1.1 over-strength factors are applied to f'_c and f_y, respectively, 1.4 otherwise.)*

l_{dv} = *Vertical development length, see Figure 31F-7-9*

D_p = *Diameter of pile*

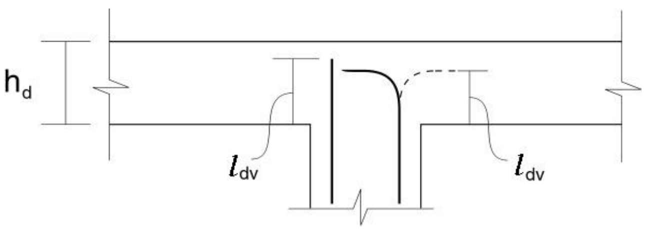


Figure 31F-7-9: Development Length

2. *Determine the nominal principal tension p_t, stress in the joint region:*

$$p_t = \frac{-f_a}{2} + \sqrt{\left(\frac{f_a}{2}\right)^2 + \nu_j^2} \qquad (7\text{-}23)$$

where:

$$f_a = \frac{N}{\left(D_p + h_d\right)^2} \qquad (7\text{-}24)$$

is the average compressive stress at the joint center caused by the pile axial compressive force N and h_d is the deck depth. Note, if the pile is subjected to axial tension under seismic load, the value of N, and f_a will be negative.

If $p_t > 5.0\sqrt{f'_c}$ psi, joint failure will occur at a lower moment than the column plastic moment capacity M_p. In this case, the maximum moment that can be developed at the pile/deck interface will be limited by the joint principal tension stress capacity, which will continue to degrade as the joint rotation increases, as shown in Figure 31F-7-10. The moment capacity of the connection at which joint failure initiates can be established from equations 7-26 and 7-27.

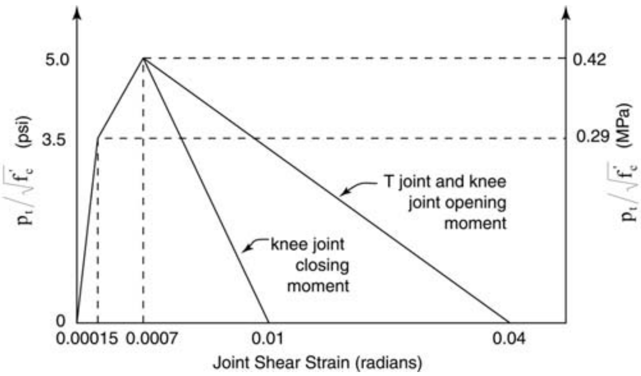


Figure 31F-7-10: Degradation of Effective Principal Tension Strength with Joint Shear Strain (rotation) [7.1, pg. 564]

For p_t = $5.0\sqrt{f'_c}$, determine the corresponding joint shear stress, ν_j:

$$\nu_j = \sqrt{p_t\left(p_t - f_a\right)} \qquad (7\text{-}25)$$

3. *The moment capacity of the connection can be approximated as:*

$$M_c = \left(\frac{1}{.90}\right)\sqrt{2}\nu_j l_{dv} D_p^2 \le M_p \qquad (7\text{-}26)$$

This will result in a reduced strength and effective stiffness for the pile in a pushover analysis. The maximum displacement capacity of the pile should be based on a drift angle of 0.04 radians.

If no mechanisms are available to provide residual strength, the moment capacity will decrease to zero as the joint shear strain increases to 0.04 radians, as shown in Figure 31F-7-11.

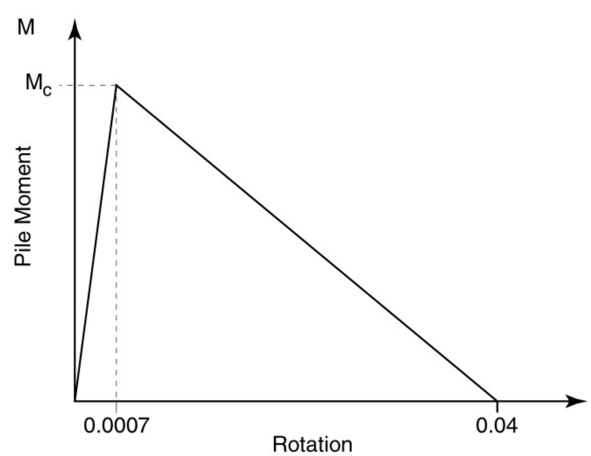


Figure 31F-7-11 Reduced Pile Moment Capacity

If deck stirrups are present within $h_d/2$ of the face of the pile, the moment capacity, $M_{c,r}$, at the maximum plastic rotation of 0.04 radians may be increased from zero to the following (see Figure 31F-7-12):

$$M_{c,r} = 2A_s f_y (h_d - d_c) + N(\frac{D_p}{2} - d_c) \qquad (7\text{-}27)$$

A_s = Area of slab stirrups on one side of joint
h_d = See Figure 31F-7-9 (deck thickness)
d_c = Depth from edge of concrete to center of main reinforcement

In addition, the bottom deck steel (A_s , deckbottom) area within $h_d/2$ of the face of the pile shall satisfy:

$$A_{s,deckbottom} \geq 0.5 \cdot A_s \qquad (7\text{-}28)$$

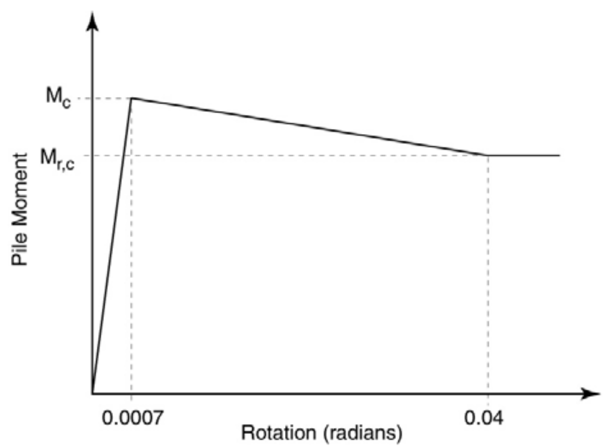


Figure 31F-7-12: Joint Rotation

4. *Using the same initial stiffness as in subsection 3107F.2.5.4, the moment-curvature relationship established for the pile top can now be adjusted to account for the joint degradation.*

The adjusted yield curvature, $\phi_u^{'}$, can be found from:

$$\phi_y^{'} = \frac{\phi_y M_c}{M_n} \qquad (7\text{-}29)$$

M_n *is defined in Figure 31F-7-5*

The plastic curvature, ϕ_p , corresponding to a joint rotation of 0.04 can be calculated as:

$$\phi_p = \frac{0.04}{L_p} \qquad (7\text{-}30)$$

Where L_p is given by equation 7-5.

The adjusted ultimate curvature, $\phi_u^{'}$, can now be calculated as:

$$\phi_u^{'} = \phi_p + \frac{\phi_y M_{c,r}}{M_n} \qquad (7\text{-}31)$$

Note that $M_{c,r}$ = 0 unless deck stirrups are present as discussed above. Examples of adjusted moment curvature relationships are shown in Figure 31F-7-13.

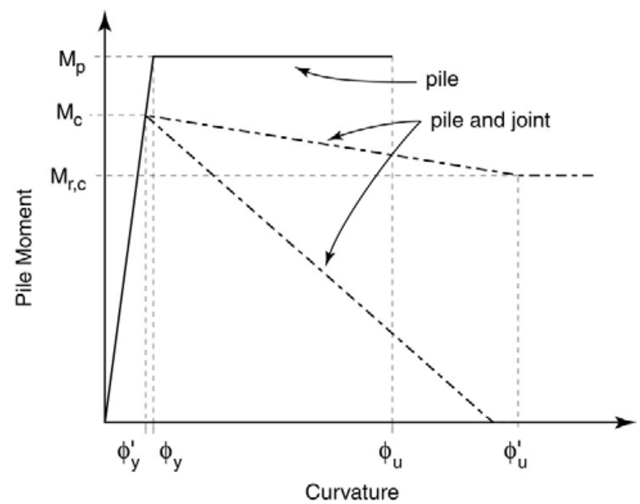


Figure 31F-7-13 Equivalent Pile Curvature

3107F.2.7.2 Development Length. *The development length, l_{dc} , is:*

$$l_{dc} \geq \frac{0.025 \cdot d_b \cdot f_{ye}}{\sqrt{f_c^{'}}} \qquad (7\text{-}32)$$

where:
d_b = *dowel bar diameter*
f_{ye} = *expected yield strength of dowel*
f'_c = *compressive strength of concrete*

In assessing existing details, actual or estimated values for f_{ye} and $f_c^{'}$ rather than nominal strength should be used in accordance with 3107F.2.1.1.

When the development length is less than that calculated by the equation 7-32, the moment capacity *shall be calculated using a proportionately reduced yield strength, $f_{ye,r}$, for the vertical pile reinforcement:*

$$f_{ye,r} = f_{ye} \cdot \frac{l_d}{l_{dc}} \qquad (7\text{-}33)$$

where:

l_d = *actual development length*

f_{ye} = *expected yield strength of dowel*

3107F.2.8 Batter Piles

3107F.2.8.1Existing Ordinary Batter Piles. *Wharves or piers with ordinary (not fused, plugged or having a seismic release mechanism) batter piles typically have a very stiff response when subjected to lateral loads in the direction of the batter. The structure often maintains most of its initial stiffness all the way to failure of the first row of batter piles. Since batter piles most likely will fail under a level 2 seismic event, the following method may be used to evaluate the post failure behavior of the wharf or pier:*

1. *Identify the failure mechanism of the batter pile-deck connection (refer to subsection 3104F.4.7) for typical failure scenarios) and the corresponding lateral displacement.*
2. *Release the lateral load between the batter pile and the deck when the lateral failure displacement is reached.*
3. *Push on the structure until subsequent failure(s) have been identified.*

As an example, following these steps will result in a force-displacement (pushover) curve similar to the one shown in Figure 31F-7-14 for a wharf supported by one row of batter piles.

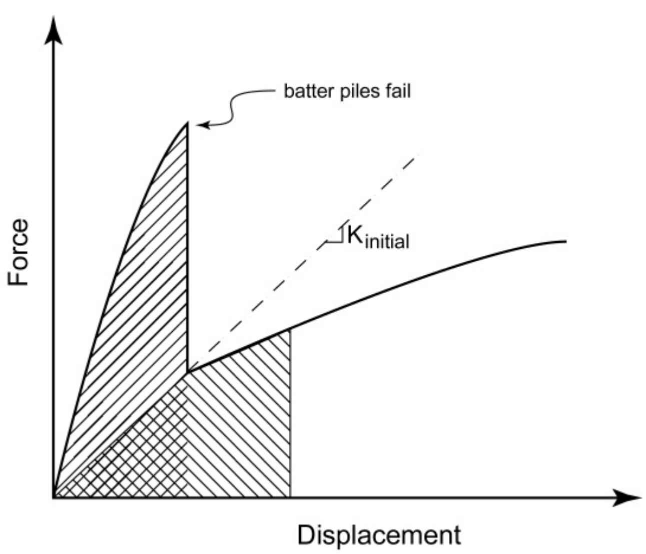


Figure 31F-7-14: Pushover Curve for Ordinary Batter Piles

When the row of batter piles fail in tension or shear, stored energy will be released. The structure will therefore experience a lateral displacement demand following the non-ductile pile failures. If the structure can respond to this displacement demand without exceeding other structural limitations, it may be assumed that the structure is stable and will start to respond to further shaking with a much longer period and corresponding lower seismic demands. The wharf structure may therefore be able to sustain larger seismic demands following the loss of the batter piles than before the loss of pile capacity, because of a much softer seismic response.

The area under the pushover curve before the batter pile failures is compared to the equivalent area under the post failure pushover curve (refer to Figure 31F-7-14). If no other structural limitations are reached with the new displacement demand, it is assumed that the structure is capable of absorbing the energy. It should be noted that even though the shear failure is non-ductile, it is expected that energy will be absorbed and the damping will increase during the damage of the piles. The above method is, therefore, considered conservative.

Following the shear failure of a batter pile row, the period of the structure increases such that equal displacement can be assumed when estimating the post-failure displacement demand. The new period may be estimated from the initial stiffness of the post failure system as shown in Figure 31F-7-14. A new displacement demand can then be calculated in accordance with subsection 3104F.2.

3107F.2.8.2 Non-ordinary Batter Piles. *For the case of a plugged batter pile system, an appropriate displacement force relationship considering plug friction may be used in modeling the structural system.*

For fused and seismic release mechanism batter pile systems, a non-linear modeling procedure shall be used and peer reviewed (subsection 3101F.6.1).

3107F.2.9 Concrete Pile Caps with Concrete Deck

3107F.2.9.1 General. *The moment-curvature and moment-rotation relationships shall be computed for pile caps using the methodology previously described. When the deck and the pile cap behave monolithically, an appropriate width of the deck may be included as part of the pile cap cross-section as per ACI-318 [7.5].*

3107F.2.9.2 Plastic Hinge Length. *The plastic hinge length L_P, for existing pile caps may be taken as:*

$$L_P = 0.5 D_c \qquad (7\text{-}34)$$

where D_c is the pile cap depth.

3107F.2.9.3 Ultimate Concrete and Steel Flexural Strains. *The ultimate strain limits defined in subsection 3107F.2.5.5 shall also apply to pile caps and deck.*

All concrete shall be treated as unconfined concrete unless it can be demonstrated that adequate confinement steel is present.

3107F.2.9.4 Component Acceptance/Damage Criteria. *For new pile caps and deck, Level 1 seismic performance shall utilize the design methods in ACI-318 [7.5]; Level 2 seismic performance shall be limited to the following strains:*

Deck/pile cap:	$\varepsilon_c \leq 0.005$
Reinforcing steel tension strain:	$\varepsilon_s \leq 0.01$

For existing pile caps and deck, the limiting strain values are defined in Table 31F-7-5.

Concrete components for all non-seismic loading combinations shall be in accordance with ACI 318 [7.5].

3107F.2.9.5 Shear Capacity (Strength). *Shear capacity shall be based on nominal material strengths; reduction factors shall be in accordance with ACI 318 [7.5].*

3107F.2.10 Concrete Detailing. *For new MOTs, the required development splice length, cover and detailing shall conform to ACI 318 [7.5], with the following exceptions:*

1. *For pile/deck dowels, the development length may be calculated in accordance with subsection 3107F.2.7.2.*
2. *The minimum concrete cover for prestressed concrete piles shall be three inches, unless corrosion inhibitors are used, in which case a cover of two-and-one-half inches is acceptable.*
3. *The minimum concrete cover for wharf beams and slabs, and all concrete placed against soil shall be three inches, except for headed reinforcing bars (pile dowels or shear stirrups) the cover may be reduced to two-and-one-half inch cover at the top surface only. If corrosion inhibitors are used, a cover of two-and-one-half inches is acceptable.*

3107F.3 Timber Piles and Deck Components

3107F.3.1 Component Strength. *The following parameters shall be established in order to assess component strength:*

New and existing components:

1. *Modulus of rupture*
2. *Modulus of elasticity*
3. *Type and grade of timber*

Existing components only:

1. *Original cross-section shape and physical dimensions*
2. *Location and dimension of braced frames*
3. *Current physical condition of members including visible deformation*
4. *Degradation may include environmental effects (e.g., decay, splitting, fire damage, biological and chemical attack) including its effect on the moment of inertia, I*
5. *Loading and displacement effects (e.g., overload, damage from earthquakes, crushing and twisting)*

Subsection 3104F.2.2 discusses existing material properties. At a minimum, the type and grade of wood shall be established. The published stress values in the ANSI/AF&PA NDS [7.9] may be used as default values and shall be multiplied by a factor of 2.8 to convert from allowable stress levels to yield or ultimate values for seismic loading.

For deck components, the allowable stresses shall be limited to the values published in the ANSI/AF&PA NDS [7.9] increased by a factor of 2.0. Piling deformation limits shall be calculated based on the strain limits in accordance with subsection 3107F.3.3.3.

The values shown in the ANSI/AF&PA NDS [7.9] are not developed specifically for MOTs and can be used as default properties only if as-built information is not available, the member is not damaged and testing is not performed. To account for the inherent uncertainty in establishing component capacities for existing structures with limited knowledge about the actual material properties, a reduction (knowledge) factor of k = 0.75 shall be included in the component strength and deformation capacity analyses in accordance with subsection 3107F.2.1.2.

The modulus of elasticity shall be based on tests or the ANSI/AF&PA NDS [7.9]. Alternatively the values shown in Table 31F-7-7 may be used for typical timber piles:

TABLE 31F-7-7

MODULUS OF ELASTICITY (E) FOR TYPICAL TIMBER PILES

Species	E *(psi)*
Pacific Coast Douglas Fir	*1,500,000*
Red Oak	*1,250,000*
Red Pine	*1,280,000*
Southern Pine	*1,500,000*

3107F.3.2 Deformation Capacity of Flexural Members. *The displacement demand and capacity of existing timber structures may be established per subsection 3104F.2.*

The soil spring requirements for the lateral pile analysis shall be in accordance with Section 3106F.

A linear curvature distribution may be assumed along the full length of a timber pile.

The displacement capacity of a timber pile can then be established per subsection 3107F.3.3.2.

3107F.3.3 Timber Piles

3107F.3.3.1 Stability. *Subsection 3107F.2.5.2 shall apply to timber piles.*

3107F.3.3.2 Displacement Capacity. *A distinction shall be made between a pier-type pile, with a long unsupported length and a wharf-landside-type pile with a short unsupported length between the deck and soil. The effective length, L, is the distance between the pinned deck/pile connection and in-ground fixity as shown in Figure 31F-7-15. For pier-type (long unsupported length) vertical piles, two simplified procedures to determine fixity or displacement capacity are described in MIL-HDBK-1025/6 [7.10] or the Navy Design Manual 7.02 [7.11], respectively.*

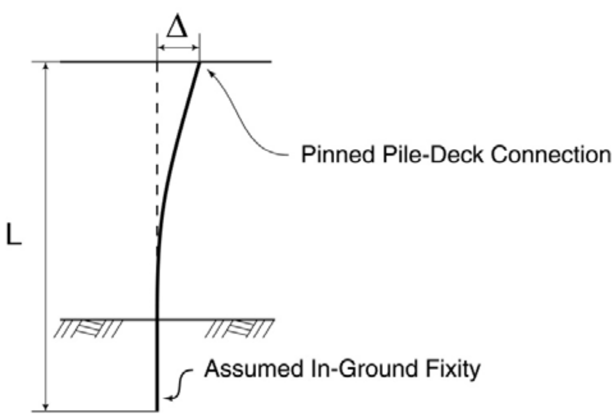


Figure 31F-7-15: Assumed In-Ground Fixity

In order to determine fixity in soft soils, another alternative is to use Table 31F-7-8.

The displacement capacity, Δ, for a pile pinned at the top, with effective length, L, and moment, M, using Table 31F-7-8 or MIL-HDBK-1025/6 [7.10] is:

$$\Delta = \frac{ML^2}{3EI} \quad (7\text{-}35)$$

where

E = *Modulus of elasticity*
I = *Moment of inertia*

Assuming linear curvature distribution along the pile, the allowable curvature, ϕ_a, can be established from:

$$\phi_a = \frac{\varepsilon_a}{c_u} \quad (7\text{-}36)$$

where:

ε_a = *allowable strain limit according to subsection 3107F.3.3.3*
c_u = *distance to neutral axis which can be taken as $D_p/2$, where D_p is the diameter of the pile*

The curvature is defined as:

$$\phi = \frac{M}{EI} \quad (7\text{-}37)$$

The maximum allowable moment therefore becomes:

$$M = \frac{2\varepsilon_a}{D_p} EI \quad (7\text{-}38)$$

The displacement capacity is therefore given by:

$$\Delta = \frac{2\varepsilon_a L^2}{3D_p} \quad (7\text{-}39)$$

TABLE 31F-7- 8

DISTANCE BELOW GROUND TO POINT OF FIXITY

Pile EI_g	*Soft Clays*	*Loose Granular & Medium Clays*
< 10^{10} lb in^2	*10 feet*	*8 feet*
> 10^{10} lb in^2	*12 feet*	*10 feet*

3107F.3.3.3 Component Acceptance/Damage Criteria. *The following limiting strain values apply for each seismic performance level for existing structures:*

Earthquake Level	Max. Timber Strain
Level 1	0.004
Level 2	0.008

Alternatively, ANSI/AF&PA NDS [7.9] may be used.

Timber components for all non-seismic loading combinations shall be designed in accordance with ANSI/AF&PA NDS [7.9].

3107F.3.3.4 Shear Capacity. *To account for material strength uncertainties, the maximum shear demand, $V_{max,push}$, established from the single pile lateral analysis shall be multiplied by 1.2:*

$$V_{design} = 1.2 V_{max,push} \quad (7\text{-}40)$$

The maximum shear stress $\tau_{\max}$, in a circular pile can then be determined:

$$\tau_{\max} = \frac{10}{9}\frac{V_{\max,push}}{\pi \cdot r^2} \qquad (7\text{-}41)$$

where:

r = *radius of pile*

For the seismic load combinations, the maximum allowable shear stress, $\tau_{capacity}$, is the design shear strength, τ_{design}, from the ANSI/AF&PA NDS [7.9] multiplied by a factor of 2.8.

$$\tau_{capacity} = 2.8\tau_{design} \qquad (7\text{-}42)$$

The shear capacity must be greater than the maximum demand.

3107F.4 Mooring and Berthing Components. *Mooring components include bitts, bollards, cleats, pelican hooks, capstans, mooring dolphins and quick release hooks.*

Berthing components include fender piles and fenders, which may be camels, fender panels, or wales.

Applicable safety factors to be applied to the demand are provided in subsection 3103F.10.

3107F.4.1 Component Strength. *The following parameters shall be established in order to calculate component strength:*

New and existing components:

1. *Yield and tensile strength of structural steel*
2. *Structural steel modulus of elasticity*
3. *Yield and tensile strength of bolts*
4. *Concrete infill compressive strength*
5. *Concrete infill modulus of elasticity*

Additional parameters for existing components:

1. *Condition of steel including corrosion*
2. *Effective cross-sectional areas*
3. *Condition of embedment material such as concrete slab or timber deck*

3107F.4.2 Mooring and Berthing Component Demand. *The maximum mooring line forces (demand) shall be established per Section 3105F. Multiple lines may be attached to the mooring component at varying horizontal and vertical angles. Mooring components shall therefore be checked for all the mooring analysis load cases. The maximum demand on breasting dolphins and fender piles shall be established according to subsection 3103F.6 and Section 3105F.*

3107F.4.3 Capacity of Mooring and Berthing Components. *The structural and connection capacity of mooring components bolted to the deck shall be established in accordance with AISC [7.8], ACI-318 [7.5], ANSI/AF&PA NDS [7.9] as appropriate. The mooring component capacity may be governed by the strength of the deck material. Therefore, a check of the deck capacity to withstand mooring component loads shall be performed.*

3107F.5 Symbols

A_e	=	*Effective shear area*
A_g	=	*Uncracked, gross section area*
A_h	=	*Total area of transverse reinforcement, parallel to direction of applied shear cut by an inclined shear crack*
A_s	=	*Area of reinforcing steel*
A_{sp}	=	*Spiral or hoop cross section area*
c	=	*Depth from extreme compression fiber to neutral axis at flexural strength*
c_o	=	*Outside of steel pipe to center of hoop or spiral or concrete cover to center of hoop or spiral*
c_u	=	*Value of neutral axis depth at ultimate strength of section*
D	=	*Pile diameter*
D^*	=	*Reference diameter of 6 ft*
d_b	=	*Dowel bar diameter*
d_c	=	*Depth from edge of concrete to center of reinforcement*
d_{bl}	=	*Diameter of the longitudinal reinforcement*
D_c	=	*Depth of pile cap*
D_p	=	*Pile diameter or gross depth (in case of a rectangular pile with spiral confinement)*
e	=	*Eccentricity of axial load*
ε_a	=	*Allowable strain limit*
ε_{cm}	=	*Max extreme fiber compression strain*
ε_{cu}	=	*Ultimate concrete compressive strain*
ε_{sm}	=	*Strain at peak stress of confining reinforcement*
ε_u	=	*Ultimate steel strain*
E	=	*Modulus of elasticity*
f'_c	=	*Concrete compression strength*
f'_{cc}	=	*Confined strength of concrete*
F_p	=	*Prestress compression force in pile*
f_p	=	*Yield strength of prestress strands*

f_y = *Yield strength of steel*

f_{ye} = *Design yield strength of longitudinal or dowel reinforcement (ksi)*

f_{yh} = *Yield stress of confining steel*

f_{yh} = *Yield strength of transverse or hoop reinforcement*

$f_{y,shell}$ = *Yield strength of steel shell*

$f_{ye,r}$ = *Reduced dowel yield strength*

h = *Width of pile in considered direction*

h_d = *Deck depth*

H = *Distance from ground to pile point of contraflexure*

I_c = *Moment of Inertia of uncracked section*

I_e = *Effective moment of inertia*

I_g = *Gross moment of inertia*

K = *Subgrade modulus*

k = *Factor dependent on the curvature ductility* $\mu_\phi = \phi / \phi_y$ *, within the plastic hinge region*

k = *Knowledge factor*

L = *The distance from the critical section of the plastic hinge to the point of contraflexure*

L_p = *Plastic hinge length*

l_{dc} = *Minimum development length*

l_d = *Existing development length*

l_{dv} = *Vertical development length*

M_c = *Moment capacity of the connection*

$M_{c,r}$ = *Moment capacity at plastic rotation*

M_n = *Moment at secant stiffness*

M_p = *As determined from a pushover analysis at displacements corresponding to the damage control limit state*

M_y = *Moment at first yield*

N = *Pile axial compressive force*

N_u = *External axial compression on pile including load due to earthquake action*

ρ_s = *Effective volume ratio of confining steel*

ρ_t = *Nominal principal tension*

r = *Radius of circular pile*

s = *Spacing of hoops or spiral along the pile axis*

t = *Shell thickness*

Δ = *Displacement*

Φ = *1.0 for existing structures, and 0.85 for new design*

θ = *Angle of critical crack to the pile axis (taken as 30° for existing structures, and 35° for new design)*

θ_p = *Plastic rotation*

α = *Angle between line joining centers of flexural compression in the deck/pile and in-ground hinges, and the pile axis*

ϕ_a = *Allowable curvature*

ϕ_m = *Maximum curvature*

ϕ_p = *Plastic curvature*

ϕ_u = *Ultimate curvature*

$\phi`_u$ = *Adjusted ultimate curvature*

ϕ_y = *Yield curvature*

$\phi`_y$ = *Adjusted yield curvature*

τ_{max} = *Maximum shear stress*

V_c = *Concrete shear strength*

v_j = *Joint shear stress*

V_{design} = *Design shear strength*

$V_{max,push}$ = *Maximum shear demand*

V_n = *Nominal shear strength*

V_s = *Transverse reinforcement shear capacity (strength)*

V_{shell} = *Shear capacity for steel pipe*

3107F.6 References

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Authority: Sections 8755 and 8757, Public Resources Code.

Reference: Sections 8750, 8751, 8755 and 8757, Public Resources Code.